FOR CONTRACT NO.: 11-253504

INFORMATION HANDOUT

MATERIALS INFORMATION

CAMINO DE LAS PALMAS FINAL FOUNDATION REPORT

CAMINO DE LAS PALMAS RETAINING WALL FOUNDATION RECOMMENDATION

CAMINO DE LAS PALMAS FOUNDATION REVIEW

ROUTE: 11-SD-125-19.4/22.4

FINAL FOUNDATION REPORT
CAMINO DE LAS PALMAS
RETAINING WALL
STATE ROUTE 125
KP 19.47 TO 22.37
SAN DIEGO COUNTY, CALIFORNIA
CALTRANS FILE NO. 11-SD-125
CU 1120 E.A. 253501

PREPARED FOR:

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May 14, 2003
Project No. 103084005

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Subject: Final Foundation Report
Camino De Las Palmas Retaining Wall
State Route 125, K.P.19.47 to 22.37
San Diego County, California

Dear Mr. Fitterer:

In accordance with your request and authorization, we are submitting this foundation report for the proposed Camino De Las Palmas retaining wall located along State Route 125 in San Diego County, California. This report presents the findings of our subsurface evaluation, which was performed to provide geotechnical criteria for the design and construction of the proposed work. Our conclusions and recommendations are presented herein.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report, we will be pleased to meet with you at your convenience.

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

In accordance with your request and authorization, we have performed a supplemental geotechnical evaluation for the proposed Camino De Las Palmas retaining wall located along State Route 125 between Kilometer Posts 19.47 and 22.37 in San Diego County, California. We previously performed a geotechnical evaluation (Ninyo & Moore, 1997) which evaluated placement of a sound berm near the top of the existing slope. The purpose of this supplemental study is to evaluate the foundation conditions for the special (non-standard) retaining wall and prepare a foundation report for its design in accordance with Caltrans guidelines (Caltrans, 2002b). This report presents our findings, conclusions, and geotechnical recommendations regarding the subject project.

2. SCOPE OF SERVICES

The scope of our geotechnical services included the following:

- Review of readily available background materials pertaining to the site, including geologic maps, stereoscopic aerial photographs, and in-house information.
- Field reconnaissance to observe the general site conditions for drilling access, to mark and select the proposed boring locations, and to coordinate with Underground Service Alert (USA) for underground utility clearance.
- Subsurface evaluation that included drilling, logging, and sampling of three small-diameter continuous flight auger borings using a limited access drill rig. The borings were drilled to depths of approximately 12.3 meters (m) below existing grade. The purpose of the borings was to evaluate the subsurface soil conditions and collect soil samples for laboratory testing.
- Laboratory testing of representative soil samples to evaluate in-situ moisture content and dry density, Atterberg limits, consolidation, shear strength, and corrosivity.
- Data compilation and engineering analyses of the information obtained from our background review, subsurface evaluation, and laboratory testing. Our engineering analyses included analysis of seismic design criteria, potential for liquefaction, design earth pressures, corrosion potential, and design criteria for foundations and retaining structures.
- Preparation of this report, including a Log of Test Boring (LOTB) sheet, presenting the results of our field exploration, laboratory testing, and engineering analyses, as well as our conclusions and recommendations relative to the geotechnical aspects of project design and construction.

3. PROJECT DESCRIPTION

Based on the preliminary plans provided by Dokken Engineering, we understand that the special (non-standard) retaining wall will be constructed from approximate wall Station 17+01 to the north to terminate at approximate wall Station 17+67 near Palm Street. The wall will be 4.8 to 7.3 m high and supported on Cast in Drilled Hole (CIDH) pile with tiebacks installed through the pile cap to provide additional sliding resistance. A Type 1 SW wall will be constructed from approximate wall Station 16+27 to approximate wall Station 17+01. A sound wall will be constructed on top of the retaining walls.

4. SITE DESCRIPTION

The proposed retaining wall and sound attenuation system is to be located near or on the crest of the east facing fill slope located along the eastern boundary of the Camino De Las Palmas residential subdivision. The slope, originally constructed at an inclination of approximately 1:1.5 (vertical:horizontal), varies in height up to approximately 8 m. At present, the slope is of variable inclination with some areas being essentially at the "as graded" inclination. However, several of the homeowners have reconfigured the slope by adding improvements such as concrete block retaining walls or terraces for planting and other uses. Although an evaluation of the condition of these backyard improvements was beyond the scope of our study, some indications of distress were noted, including cracking and possible tilting of retaining structures. The distress may have been due to a combination of several possible factors, including soil movement caused by slope creep, consolidation or expansive soils, as well as the action of tree roots. If desired, we could perform a detailed evaluation of the existing conditions at the residential properties. In addition, areas of minor slope wash were also noted, particularly on slopes/yards with little of no vegetation. Where vegetation is present, it generally consisted of palm trees, small fruit trees and landscaping ground cover.

5. SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface evaluation consisted of excavating three small-diameter, continuous flight auger borings. The locations of the borings were selected based on our understanding of the project and approximately located in the field by measuring off distances from existing improvements. The approximate locations of the borings are indicated on the LOTB Sheet (Sheet 1).

5.1. Subsurface Evaluation

Three continuous flight auger borings were conducted on December 12, 2002. A limited access, continuous flight auger drill rig was used to excavate the exploratory borings to depths of up to approximately 12.3 m below existing grade. A representative of our firm logged the borings. The purpose of the borings was to evaluate the subsurface soil conditions and collect soil samples at selected depths for laboratory testing. The type of samples collected included bulk samples and relatively undisturbed samples utilizing the California modified split-spoon sampler. Logs of the borings are presented on the LOTB Sheet (Sheet 1).

5.2. Laboratory Testing

Laboratory testing of selected soil samples included in-situ moisture content and dry density, Atterberg limits, consolidation, shear strength, and corrosivity (soil pH, minimum electrical resistivity, water-soluble sulfate content, and water-soluble chloride). The results of the moisture content and dry density tests are recorded on the LOTB Sheets (Sheet 1). The other laboratory test results are presented in Appendix A.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The following sections describe geologic, soil, and groundwater conditions at the site. Faulting, seismicity, and liquefaction potential at the site are also addressed.

6.1. Regional Geologic Setting

The project area is situated in the western San Diego County section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in San Diego County, in which the project area is situated, generally consists of sedimentary rocks of the Eocene-age Sweetwater and Mission Valley Formations.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are considered active faults. The Whittier-Elsinore, San Jacinto, and San Andreas faults are active fault systems located northeast of the project area and the Newport Inglewood-Rose Canyon and Palos Verdes-Coronado Bank faults are active faults located north and west of the project area (Figure 2). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

6.2. Site Geology

Based on our geologic reconnaissance, our subsurface exploration, review of published geologic maps and stereoscopic aerial photographs, the soil profile present at the project site consists of surficial soils of fill and colluvium underlain by materials of the Sweetwater and Mission Valley Formations to the maximum depths explored. The earth materials encountered in the project area are described in the following sections.

6.2.1. Fill

Fill soil was encountered in the borings to depths of from approximately 0.6 m to 1.2 m below existing grade. The fill primarily consisted of damp to moist, stiff to very stiff, silty clay. The fill is generally associated with the construction of the adjacent residential properties or homeowner improvements.

6.2.2. Colluvium

Colluvial soils were encountered below the fill soils to depths of approximately 2.1 m to 5.8 m below existing grade. The colluvial soils generally consisted of damp to moist, stiff to very stiff, silty clay.

6.2.3. Sweetwater Formation

Materials associated with the Sweetwater Formation were encountered in our borings below the colluvium. The materials of the Sweetwater Formation generally consisted of damp, moderately to strongly indurated silty claystone to clayey siltstone, and moderately cemented fine-grained sandy siltstone.

6.2.4. Mission Valley Formation

Materials associated with the Mission Valley Formation were encountered below materials of the Sweetwater Formation to the maximum depths explored in the borings. The Mission Valley formational materials consisted primarily of damp, weakly to moderately cemented silty fine-grained sandstone.

6.3. Groundwater

Groundwater was not encountered in our exploratory borings. Based on our experience in the vicinity of the project site, we anticipate that groundwater is at a depth greater than 20 m. It should be noted that groundwater levels are influenced by seasonal variations in rainfall, local irrigation, and groundwater pumping, and are, therefore, subject to variation.

7. FAULTING AND SEISMICITY

The site is considered to be in a seismically active area, as is much of southern California. Based on our review of stereoscopic aerial photographs and pertinent geologic literature, and our limited geologic field reconnaissance, no active faults are known to cross the project site. Table 1 presents a summary of selected major faults in the area, distance to these faults, the type of fault displacement, and the maximum moment magnitude of the fault.

Table 1 – Seismic/Fault Parameters

Fault	Approximate Fault to Site Distance kilometers[1]	Type of Displacement[2]	Maximum Moment Magnitude[2]
Newport Inglewood-Rose Canyon	11	ST	7.0
Coronado Bank	24	ST	7.8
Whittier-Elsinore	31	ST	7.5
San Jacinto	52	ST	7.5

Notes:
[1] Mualchin, 1996a
[2] Mualchin, 1996b
ST = Strike Slip

Seismic hazards at the site can be attributed to ground shaking resulting from events on active faults. In general, seismic hazards might include strong ground motion, ground surface rupture, liquefaction, and damage caused by seismically induced settlement and lateral spreading. These potential hazards are discussed in the following sections.

7.1. Strong Ground Motion

In our opinion, the design seismic event with respect to the proposed retaining wall should be an earthquake associated with the Rose Canyon fault zone located west of the project site. The California Seismic Hazard Map (Mualchin, 1996a and 1996b) indicates that the retaining wall site is located approximately within the 0.4g contour.

7.2. Surface Fault Rupture

Surface fault rupture is the offset or rupturing of the ground surface by relative displacement across a fault during an earthquake. No active or potentially active faults are known to underlie the project site; therefore, the potential for surface fault rupture is considered to be low.

7.3. Liquefaction

Liquefaction of soils can be caused by relatively strong vibratory motion due to earthquakes. Research and historical data indicate that loose, granular soils with silt and clay contents of less than 35 percent or non-plastic silt are susceptible to liquefaction and dynamic settlement, while the stability of the majority of clayey silts, silty clays, and clays is not adversely affected by vibratory motion. Based on the relatively dense and cohesive nature of the subsurface materials encountered in our exploratory borings, and the lack of a shallow groundwater table, it is our opinion that the potential for liquefaction at the site is nil. Therefore, dynamic settlement due to liquefaction was not evaluated and is not considered to be a concern.

8. ANALYSIS

The following sections describe our method of analysis for each of the structure types addressed in this report.

8.1. Modified Type 1 SW Retaining Wall

The analyses of the retaining wall foundations included the evaluation of several different types of foundations, including continuous footings, driven piles, and CIDH piles. Based on our analyses and our understanding of the project, it is our opinion that the retaining walls may be supported on continuous footings founded in competent embankment fill, recompacted fill, or in formational material. Based on preliminary wall plans, the bottom of the footing will be at elevations of approximately 123.6 to 125.6 m mean sea level (MSL).

8.2. Modified Type 7 Retaining Wall

Our analyses assumed the wall would be supported on a concrete pile cap founded on two rows of 750-millimeter (mm) diameter CIDH piles. One row of tiebacks will be installed through the pile cap to provide additional sliding resistance. It is anticipated that the tiebacks will be inclined downward at approximately 30 degrees from the horizontal. We recommend that the bearing plates be installed and the tiebacks be locked off at the design force. Design and construction of the project will be in accordance with Caltrans standards. Tieback anchorage is anticipated to be accomplished within formational material. Based on preliminary wall plans, the bottom of the pile cap will be at elevations of approximately 123.3 to 124.6 m MSL.

9. RECOMMENDATIONS

Based on the results of our subsurface evaluation and our understanding of the proposed construction, we present the following recommendations relative to the geotechnical aspects of construction of the proposed retaining walls.

9.1. Material Properties

Strength parameters for analysis of spread footings CIDH pile and global slope stability were obtained from laboratory test results and our professional experience. The material properties of the embankment fill, colluvium, and formational materials used in the analysis are presented in Table 2.

Table 2 – Material Properties

Material Type	Total Unit Weight (kN/m^3)	Cohesion (kPa)	Angle of Internal Friction (degrees)
Embankment Fill	18.9	9.6	32
Colluvium – Sandy Clay	19.7	19.2	26
Colluvium – Clayey Sand	18.5	5.0	37
Formational Material	20.4	35.9	36

9.2. Retaining Walls

The retaining walls are anticipated to include modified Type 1 SW and Type 7 walls. The retaining walls may be designed for static conditions using an equivalent fluid weight of 5.5 kN/m^3 for level backfill. Restrained retaining walls may be designed for an equivalent fluid weight of 8.4 kN/m^3 for level backfill. These values assume granular backfill and free-draining conditions. The retaining walls should be backfilled with material that meets the requirements for structure fill as presented in Section 19 of the Standard Specifications. The retaining walls should include free-draining backfill materials and perforated drains as designed by the project civil engineer, and should be constructed in accordance with Bridge Detail 3-5 on Plan B0-3 of the Caltrans Standard Plans.

In order to reduce potential efflorescence staining of the wall face, waterproofing the back of the walls should considered. The retaining wall footings should be designed in accordance with the foundation design recommendations presented in this report and Caltrans specifications. The wall should be reinforced in accordance with structural considerations.

9.2.1. Modified Type 1 SW Retaining Wall

We anticipate that the modified Type 1 SW retaining wall will be 1.8 to 4.8 m high and founded in competent embankment fill, recompacted fill, or in formational material. In order to provide a more uniform bearing condition, we recommend that a zone of compacted fill be constructed below the wall footing. This zone of compacted fill should extend 600 mm horizontally beyond the edges of the wall foundation and 1 m or more below the base of the wall foundation. The excavated material should be moisture conditioned placed in layers, and the layers compacted to 95 or more percent relative compaction in accordance with Caltrans Test (CT) 216. Fill should be placed and compacted in accordance with the recommendations of Section 10.1. Continuous footings founded in recompacted fill or in formational material may be designed for an ultimate bearing capacity of 600 kPa. Factors of safety for various loading conditions are presented in Table 4. Based on our analyses, settlement of the footings should generally be 13 mm or less. Most of this settlement should occur during construction.

Foundations placed in recompacted fill or formational material may be designed using a coefficient of friction of 0.50 (total frictional resistance equals coefficient of friction times the dead load). Using a factor of safety of 1.5, an allowable passive resistance value of 13 kPa per m of depth (with a maximum value of 40 kPa) may be used for ground sloping at an inclination of 1:1.75. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.2.1.1. Retaining Wall Foundation Factors of Safety

The allowable bearing capacity for various loading conditions of the retaining walls may be derived by dividing the ultimate bearing capacities presented above by the appropriate factor of safety. The recommended factors of safety are presented in Table 3.

Table 3 – Factors Of Safety - Retaining Walls

Load Type	Factor of Safety
Dead and Live Loads	3.0
Transient Loads	1.5

9.2.2. Modified Type 7 Retaining Wall

Based on our review of the proposed project plans and conversations with project civil engineers, we understand that the modified Type 7 wall will be 4.8 to 7.3 m high and supported on CIDH pile with tiebacks installed through the pile cap to provide additional sliding resistance.

9.2.2.1. Axial Capacity

The axial pile capacities for the proposed CIDH piles were analyzed using the methods of Reese and Meyerhof, for tension (side friction) and end bearing, respectively. Table 2 presents the results of our analysis for 750-mm CIDH pile. Piles were assumed to extend 2.0 m into competent formational material for a design ultimate load of 1800 kN. The ultimate axial pile capacities presented are based on end bearing only. Allowable axial capacity is equal to the ultimate capacity divided by a factor of safety of 2.0. The actual depth of embedment should be evaluated based on the design loads required by the structural engineer. In addition, a minimum pile spacing of three pile diameters on center should be maintained.

Table 4 – Axial Pile Capacity

Pile Diameter	Embedment into Formation (m)	Proposed Pile Tip Elevation[1] (m)	Allowable Load[2] (kN)	Ultimate Resistance[3]	
				Compression (kN)	Tension (kN)
750-mm	2	114.25	900	1800	80

Notes:
[1]Pile tip elevations are based on a pile cutoff elevation of 123.45 m MSL.
[2]Design loading not provided, assumed the given values according to Caltrans Standards.
[3]Ultimate compression capacity based on end bearing only. Tension is equal to 40% of side friction.

9.2.2.2. Lateral Capacity

Lateral pile capacities were analyzed using the LPILE Plus computer program (ENSOFT, 1997). Lateral capacities for the CIDH piles supporting the retaining wall, assuming lateral deflections of 6, 13, and 25 mm, at the top of the pile for both free-head and fixed-head conditions are summarized in Tables 5 and 6, respectively. Maximum moments generated by the indicated deflections are based on geotechnical considerations only. We recommend that the maximum moment capacities of the piles be evaluated by the structural engineer. Lateral capacities for pile lengths and embedment conditions that are different from those assumed in our analyses may be different than those indicated.

Table 5 – Lateral Pile Capacity – Retaining Wall
Free Head Condition

CIDH Pile Diameter (mm)	**750**		
CIDH Pile Length (m)	**9.2**		
Proposed Pile Tip Elevation (m)	**114.25**		
Allowable Deflection (mm)	**6**	**13**	**25**
Lateral Capacity (kN)	171	339	581
Max. Positive Moment (m-kN)	239	478	931
Max. Negative Moment (m-kN)	0.1	0.2	0.2
Depth to Max. Positive Moment (m)	2.4	2.4	2.5
Depth to Max. Negative Moment (m)	7.9	7.9	8.0
Depth to 1st Point of Zero Deflection (m)	4.3	4.3	4.3

Table 6 – Lateral Pile Capacity – Retaining Wall
Fixed Head Condition

CIDH Pile Diameter (mm)	**750**		
CIDH Pile Length (m)	**9.2**		
Proposed Pile Tip Elevation (m)	**114.25**		
Allowable Deflection (mm)	**6**	**13**	**25**
Lateral Capacity (kN)	448	870	1438
Max. Positive Moment (m-kN)	207	414	820
Max. Negative Moment (m-kN)	749	1483	2702
Depth to Max. Positive Moment (m)	3.8	3.8	3.9
Depth to Max. Negative Moment (m)	0	0	0
Depth to 1st Point of Zero Deflection (m)	5.9	5.9	6.0

For lateral loading, piles in a group may be considered to act individually when the center-to-center spacing is greater than 3B (where B is the diameter of the pile) in the direction normal to loading, and greater than 8B in the direction parallel to loading. Table 7 presents the lateral load reduction factors to be applied for various pile spacings for in-line loading.

Table 7 – Lateral Load Reduction Factors

Center-to-Center Pile Spacing for In-Line Loading	Ratio of Lateral Resistance of Pile in Group to Single Pile
6B	1.0
5B	0.9
4B	0.8
3B	0.7

9.2.2.3. Tiebacks

Tiebacks may consist of either multi-strand steel tendons or steel bars placed in inclined drilled holes and backfilled with low-slump concrete grout. Care should be taken to maintain a horizontal and vertical separation of 1.5 m or more between individual tiebacks. The tiebacks should be inclined between 28 and 32 degrees below horizontal, and should be between 120 and 400 mm in diameter.

We understand the design tieback tensile force for straight-shaft, drilled and grouted tieback anchors will be approximately 350 to 425 kN. Different tieback diameters and bonded lengths will result in different tieback capacities. The free anchor length will vary with tieback inclination but should not be less than 5 m.

9.2.2.4. Tieback Installation

Multi-strand tendon or bar, corrosion resistant anchors should be installed in drilled holes using centering devices to improve anchor uniformity. The anchor holes should be filled with concrete placed using tremie techniques out to the limit of the unbonded length. The unbonded length should remain ungrouted until after testing and lock-off of the anchor. Permanent anchors should be backfilled with lean-mix concrete after anchor testing. If caving occurs, the unbonded length should be backfilled with well-compacted sand or casing during testing. The sand or casing should be removed and replaced with lean-mix concrete after testing.

Based on the types of material encountered, drilling of the tiebacks is anticipated to be generally feasible with medium-duty equipment in good working order. We recommend that a qualified contractor evaluate a suitable method for excavation of the tiebacks.

9.3. Slope Stability Analysis

Slope stability analyses were performed to evaluate the global stability of the slope where the retaining walls will be constructed. The material properties of the embankment fill, colluvium, and formational materials used in the analysis are presented in Table 2. Cross sections were analyzed using the modified Janbu and modified Bishop methods using the computer slope stability program STABL6H. Circular and block failure surfaces were evaluated to identify critical surfaces. Based on our evaluation, a stability factor of safety greater than 1.5 is indicated after construction of the retaining wall. Anchorage material for tiebacks (formational material) was 1.5 m or more beyond the critical failure surfaces. Pseudo-static slope stability analyses indicated that a seismic coefficient (K_h) of 0.4 or greater corresponds to the yield condition (factor of safety of 1.0).

9.4. Corrosion Analysis

The corrosion potential of the on-site material at the site was evaluated for its effect on steel and concrete structural members. The corrosion potential was evaluated using the results of laboratory tests on samples obtained during the subsurface evaluation. Laboratory testing was performed on representative soil samples to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The pH and minimum electrical resistivity tests were performed in accordance with CT 643, and sulfate and chloride tests were performed in accordance with CT 417 and 422, respectively.

Test results indicate that the pH of the sample tested was 8.0, minimum electrical resistivity was 375 ohm-cm, chloride content was 325 parts per million (ppm), and soluble sulfate content was 0.07 percent. In accordance with Memo 3.1 of the Bridge Memo to Designers, a corrosive site is an area where the soil contains more than 500 ppm of chlorides, more than

2,000 ppm of sulfates, or has an electrical resistivity of less than 1,000 ohm-cm. Based on the minimum resistivity test results, the site is considered indicative of a corrosive area.

We recommend that 75 mm or more concrete cover be maintained over the reinforcing steel of foundations and other buried concrete. In accordance with Caltrans Design guidelines, Type II cement should be used with a water-cement ratio of 0.5 or less for structures that will be in contact with soils at the site. The concrete should have 390 kilograms (kg) or more of cement per cubic meter.

10. CONSTRUCTION CONSIDERATIONS

The following sections describe the anticipated geotechnical considerations for construction of the retaining and sound walls.

10.1. Fill Placement and Compaction

Loose or disturbed fill, or other unsuitable material encountered in foundation excavations, if any, should be excavated to competent material. The excavated materials should then be placed and compacted in accordance with Section 19 of the Caltrans Standard Specifications. Fill beneath foundations should be compacted to 95 or more percent of the maximum laboratory density, as evaluated by CT 216. Fill should be tested for specified compaction by the geotechnical consultant and/or Caltrans.

10.2. Temporary Excavations and Shoring

Temporary excavations should be constructed in accordance with Occupational Safety and Health Administration (OSHA) requirements. Temporary excavations and deeper than about 1.5 m, should be laid back at a slope no steeper than 1:1.5. The recommended slope ratio does not preclude local raveling and sloughing. Excavation slope surfaces should be kept moist to retard raveling and sloughing. Water should not be allowed to flow over the top of excavations in an uncontrolled manner. Stockpiled material and/or equipment should be kept back from the top of excavations a distance equivalent to the height of the excavation. Workmen should be protected from falling rocks, sloughing, and raveling of the cut in accordance with OSHA regulations. We recommend that excavation slopes be observed by the geotechnical consultant during excavations so that appropriate additional recommendations necessitated by actual field conditions may be provided. Temporary excavations are time sensitive and local failures are possible.

10.3. CIDH Piles and Tiebacks

The CIDH and tieback excavations should be observed by the geotechnical consultant during excavation to evaluate if they have been extended to the recommended depth or deeper. The excavations should be cleaned of loose soil and gravel. It is the Contractor's responsibility to take the necessary provisions to provide for the integrity of the excavation and to see that the excavations are cleaned and straight and that sloughed loose soil is removed from the bottom of the excavation prior to the placement of concrete. CIDH pile construction criteria are described in Section 49-4.03 of Caltrans Standard Specifications. CIDH piles should be checked for alignment and plumbness during installation. CIDH piles should not deviate from plumb more than 40 mm per 3 m of length. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile. Tieback construction criteria are described in Chapter 11 of the Caltrans Foundation Manual (1997).

Groundwater was not encountered during our subsurface exploration. As a result, dewatering is not expected. However, if during the time of construction groundwater is encountered, it is possible that water will accumulate in portions of the drilled holes for the CIDH piles and/or tiebacks, and some sloughing of the sides of the holes will occur. We recommend that the concrete be placed in a relatively dry excavation, unless special measures, such as placement of concrete by tremie method, are implemented to see that the aggregate and cement do not segregate during concrete placement.

10.4. Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, should undergo total settlements of less than about 13 mm. Differential settlements are typically limited to less than one-half the total amount. The retaining wall backfill is likely to induce some settlement within the clayey fill and colluvium, which may result in some settlement of the rear yard areas.

11. CONSTRUCTION OBSERVATION

The recommendations in this report are based on preliminary structural design information for the proposed construction and on subsurface information disclosed by our geotechnical evaluation and review of previous site evaluations. The assumed subsurface conditions should be checked in the field by the geotechnical engineer during construction. We recommend that project plans be reviewed by the geotechnical engineer, and that the geotechnical engineer observe and document the foundation excavations. As a result of the plan review and field observations, some recommendations presented in this report may be revised or modified to meet the project requirements.

12. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only and may not provide sufficient data to prepare an accurate bid by some contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

13. SELECTED REFERENCES

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AERIAL PHOTOGRAPHS				
Source	**Date**	**Flight**	**Numbers**	**Scale**
USDA	4-14-53	AXN-10M	9 and 10	1:20,000

State of California Business, Transportation and Housing Agency

Memorandum

Flex Your Power!
Be energy efficient!

To: Cory Binns
District 11
Office of Design, MS 35

Date: May 6, 2003

File: 11-SD-125
KP 19.4/22.1
11-253501

From: DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2

Subject: Camino De Las Palmas Retaining Wall- Foundation Report

In accordance with your request we have reviewed the foundation report for the subject project. The report is titled " Foundation report, Camino De Las Palmas Retaining Wall, State Route 125 KP 19.47 to 22.37, San Diego County, California." The report was prepared by Ninyo and Moore Geotechnical and Environmental Sciences Consultants and is dated March 20, 2003.

A previous draft version of this report was earlier reviewed by us and our comments were submitted to the Consultant. These comments were incorporated in the final report. Based on our review we have determined the report to be adequate and consistent with Caltrans Guidelines for Foundation Investigation Reports. The report is approved for use in the design of the subject wall.

If you have any questions or comments regarding this memorandum, please call Zia Yazdani at (858) 467-4054 (Calnet 734-4054).

Zia Yazdani
Associate Materials and
Research Engineer

ZY
cc : Bhinman
Aabghari

State of California | Business, Transportation and Housing Agency

Memorandum

Flex Your Power!
Be energy efficient!

To: Cory Binns
District 11
Office of Design, MS 35

Date: March 20, 2003

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KP 19.4/22.1
11-253501

RECEIVED

APR 11 2003

OFFICE OF STRUCTURE
EXTERNAL LIAISON & SUPPORT

From: DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2

Subject: Camino De Las Palmas Retaining Wall- Settlement Analysis

In accordance with your request we have evaluated the potential settlement that may result from construction of the proposed Camino De Las Palmas retaining wall located along State Route 125 between Kilometer Posts 19.4 and 22.1 in San Diego County, California. The retaining wall which will support a sound wall is proposed to be located at the State Right of Way Line that separates the freeway from existing residential structures on Camino De Las Palmas. This letter addresses the estimated magnitude and time rate of settlements that can be expected to occur in the rear yard areas of the existing residential structures as a result of the new fill that will be placed behind the wall.

The subject wall will be about 141m in length and will vary in height from 2.3 to 7.3 m. A 74 m section of wall will be supported on spread footing; the remaining 67 m section will be supported on a 750mm CIDH pile foundation. The height of new backfill behind the wall will be about 5m. The backfill will be placed against an existing fill slope that is essentially at a slope ratio of 1:1.5 and which is part of the rear yards of the existing residential structures. The existing fill slope is about 18m high. The residential structures are supported on the existing fill soils.The placement of wall backfill will impose additional loads on the existing fill soils and result in settlement of the rear yards. The question has arisen as to what magnitude of settlements can be anticipated and what impact these settlements might have on the existing residences and rear yard improvements.

In our estimate we have utilized data that were presented in previous geotechnical reports for the project that were prepared by Ninyo and Moore. Recent borings performed by Ninyo and Moore at the toe of the existing fill slope for design of the CIDH pile foundation indicate that the subsurface materials are comprised of stiff to very stiff, silty clay fill soils to a depth of about 0.6 to 1.2 m. The fill soils are underlain by approximately 2.1 to 5.8 m of colluvial soils consisting of stiff to very stiff, silty clay. The Sweetwater and Mission Valley formation underlie the upper fill and colluvial soils. The upper existing fill and colluvial soils are compressible and will contribute to the bulk of the expected settlements. The formational soils are deemed to be essentially incompressible.

Settlement estimates have been prepared for the loading configurations and soil profiles shown for cross-sections A-A', B-B' and C-C', respectively. These estimates are based on consolidation test data for fill and colluvium that were presented by Ninyo and Moore in their geotechnical engineering reports for the project. The results of our settlement estimates are presented in the figures that are attached to this report. Estimated total settlements for cross-section A-A' range from 25 to 35mm along the two edges of the new fill and 7mm at a point 1.5m beyond the edge of the new fill immediately adjoining the rear yard. For section B-B' estimated total settlements range from 49 to 56mm along the two edges of the new fill and 11mm at a point 1.5m beyond the edge of the new fill immediately adjoining the rear yard. For section C-C' estimated total settlements range from 19 to 42mm along the two edges of the new fill and 7mm at a point 1.5m beyond the edge of the new fill immediately adjacent to the rear yard.. Settlements are estimated to be very low to negligible from a distance of 1.5m to 3m beyond the edge of the new fill immediately adjacent to the rear yard. No settlements are expected to occur at the residential structures which are about 6 to 9m away from the edge of the proposed new fill. Additionally, it is anticipated that the estimated settlements will occur rapidly and that settlements will stabilize within 90 days on completion of fill placement.

Based on the above estimates, it can be concluded that settlements from the new fill loads will vary from about 25 to 56mm (1 to 2 inches). While these settlements are not deemed to be inordinately high, distress to patio slabs, pool decks, planter walls and other exterior improvements within 1.5 to 3m of the edge of the new fill may possibly occur from the 7 to 11mm of estimated settlements. Based on our analysis it is our opinion that the nearby residential structures should not undergo any new settlement as a result of the construction of the wall.

We recommend that floor level surveys of the interiors of the residential structures be performed prior to and three months after completion of the wall backfilling operations. This should be done in order to establish whether any alleged distress could be attributed to construction of the retaining wall. Additionally, we recommend that survey points be established at random locations in the rear yards and also the residential structures to monitor any potential movements as a result of the construction.

If you have any questions or comments regarding this memorandum, please call Zia Yazdani at (858) 467-4054 (Calnet 734-4054.

Zia Yazdani

Zia Yazdani
Associate Materials and
Research engineer

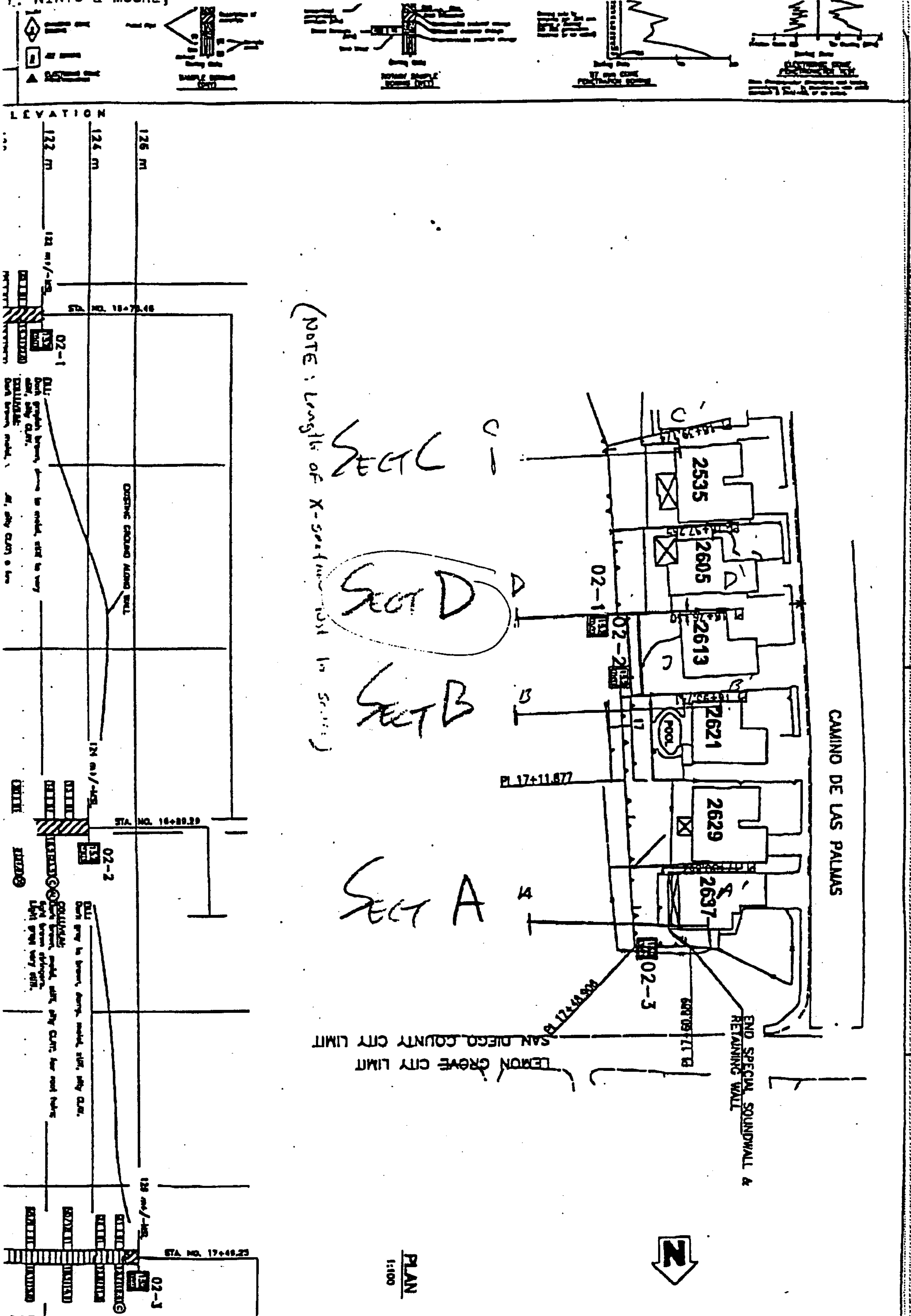
REGISTERED PROFESSIONAL ENGINEER
ZIAULLAH YAZDANI
No. 2119
Exp. 3-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

ATTACHMENTS

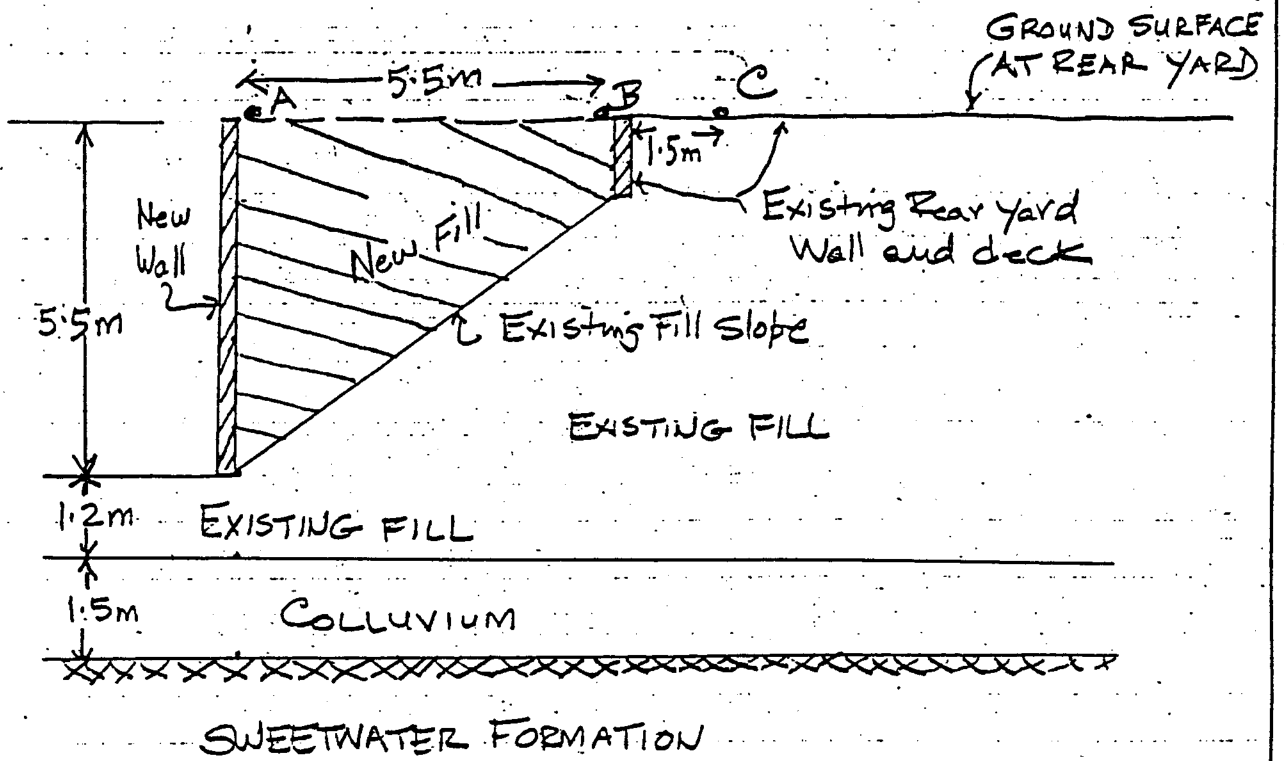
1. Cross-sections showing Settlement Estimates.

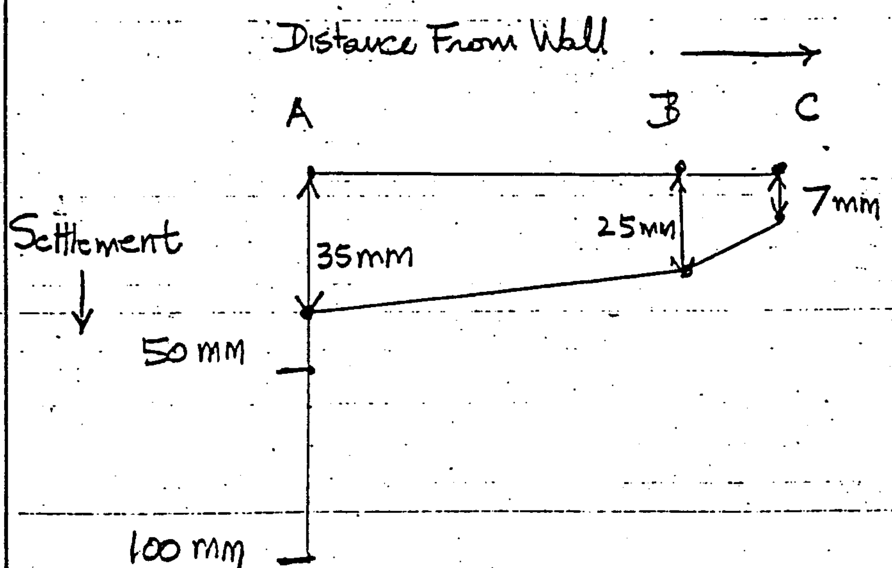
ZY

cc:
BHinman
AAbghari
RGES 02


ELEVATION
122 m
124 m
126 m
STA. NO. 16+89.29
STA. NO. 17+49.25
02-1
02-2
02-3
SECT C
SECT D
SECT B
SECT A
C
C'
D
D'
B
A
A'
2535
2605
2613
2621
2629
2637
POOL
PL 17+11.677
CAMINO DE LAS PALMAS
SAN DIEGO COUNTY CITY LIMIT
LEMON GROVE CITY LIMIT
END SPECIAL SOUNDWALL & RETAINING WALL
PLAN
1:100
N

SECTION A-A'





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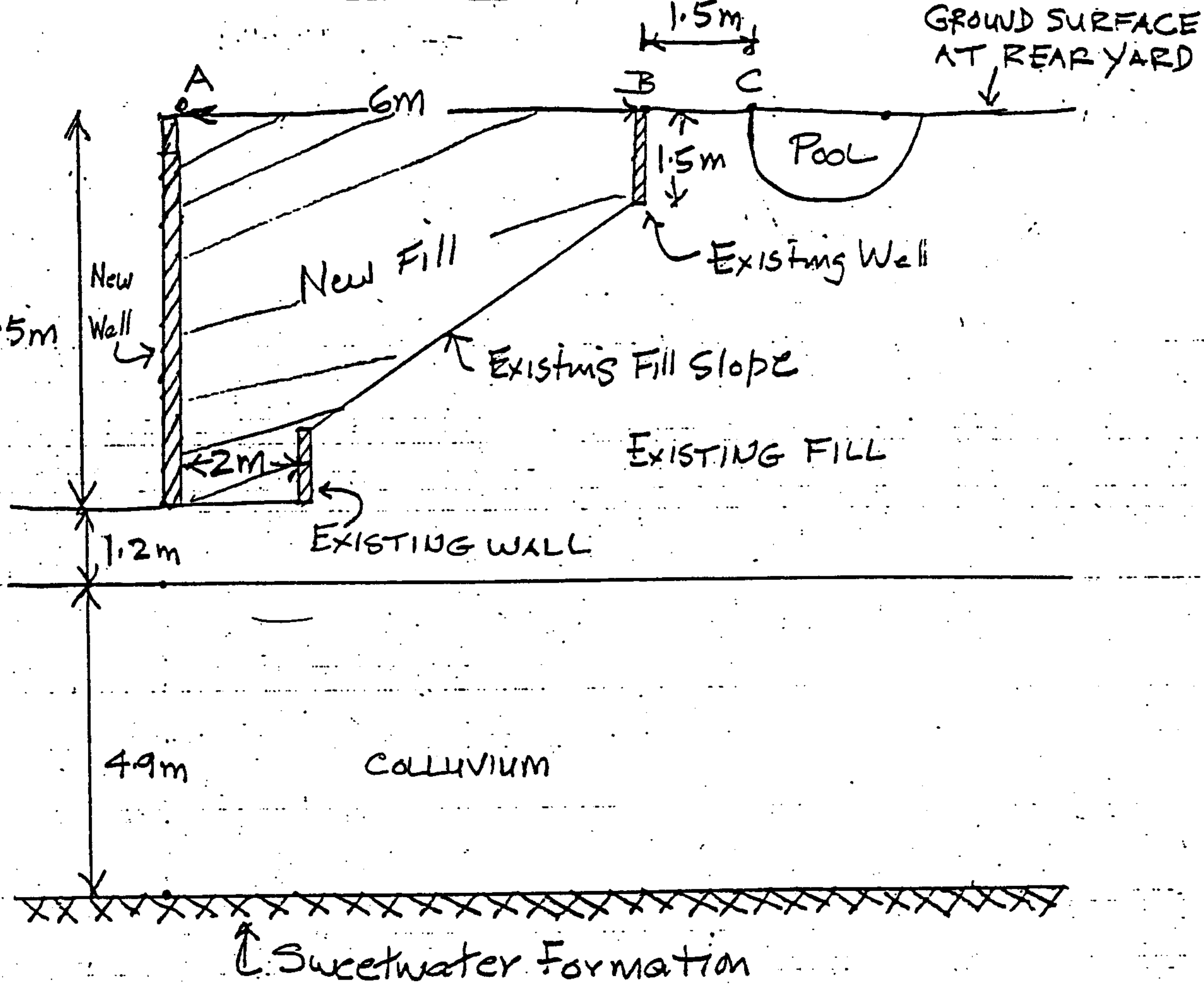
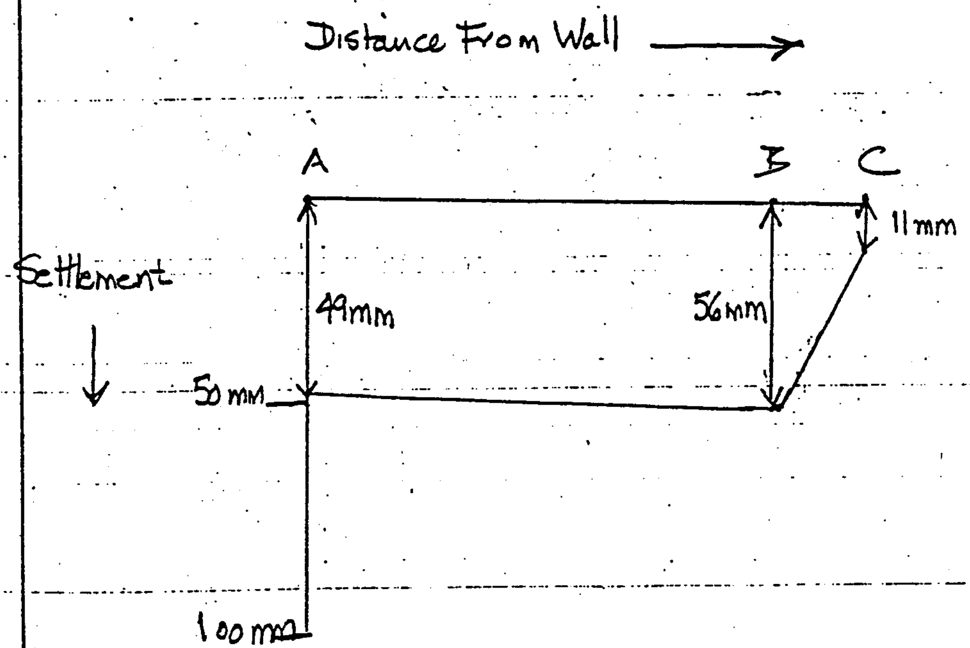
SECTION B-B'





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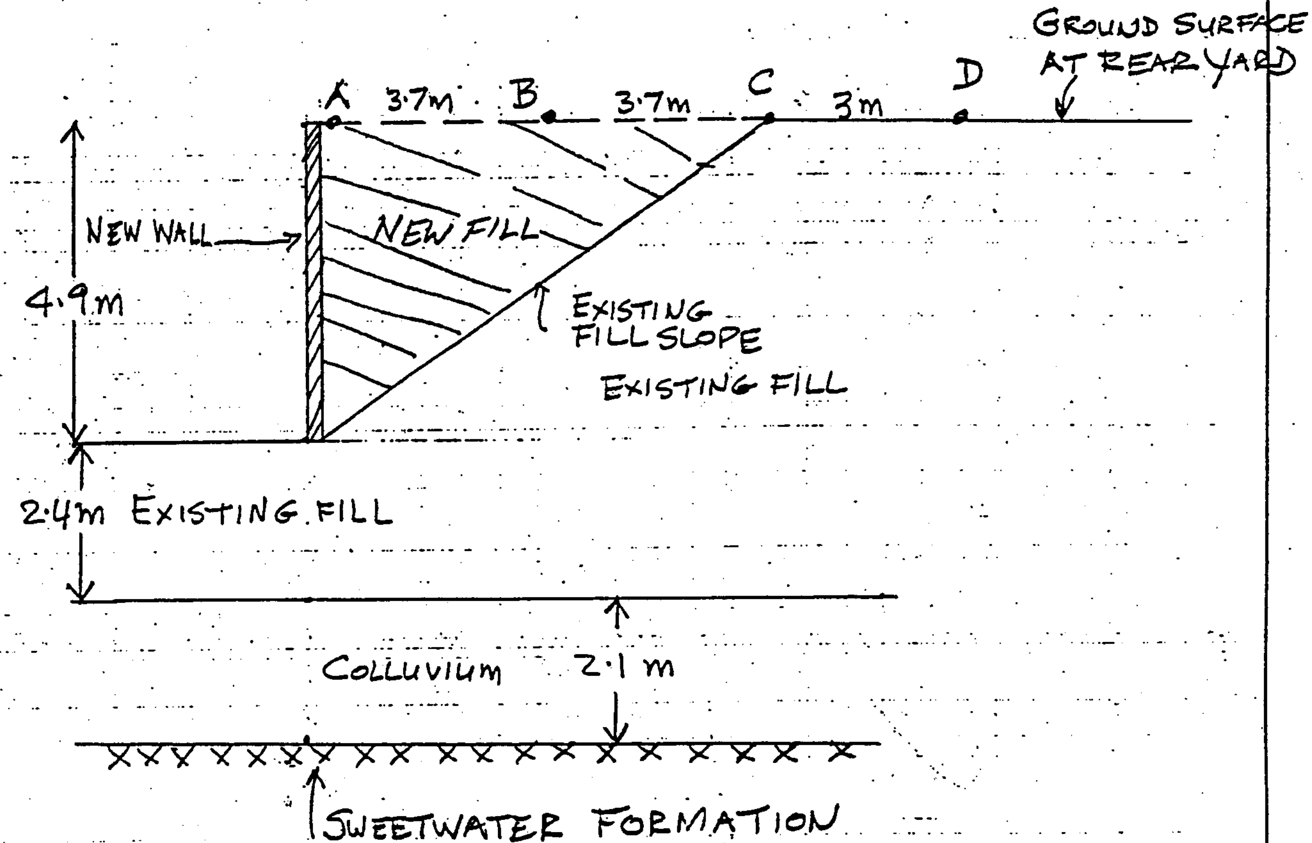
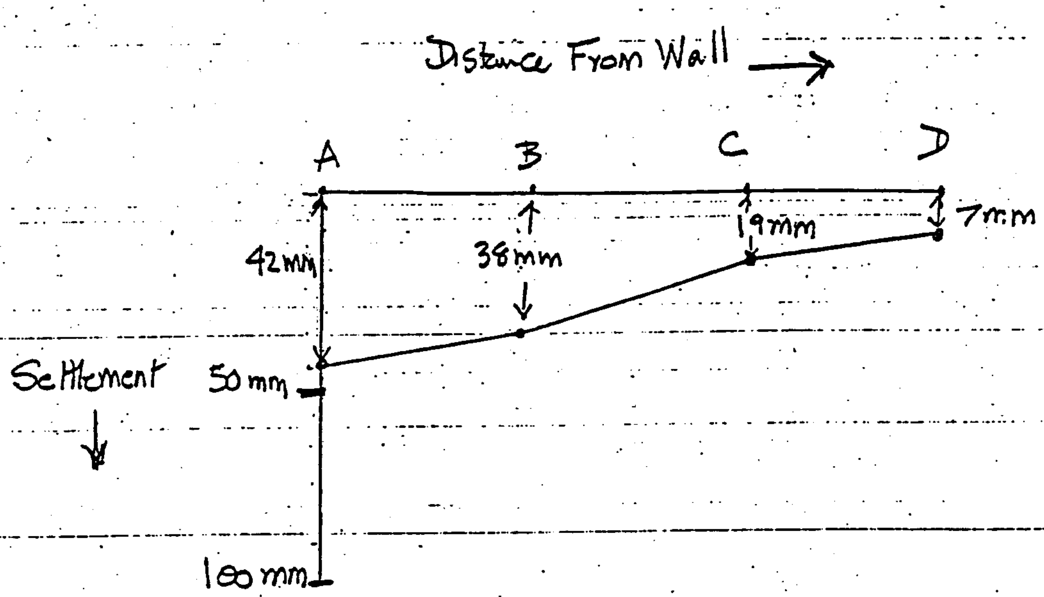
SECTION C-C'





NTS